Improving the Foundation Layers for Concrete Pavements

TECHNICAL REPORT:
Field Assessment of Jointed Portland Cement Concrete Pavement with Premature Distresses – Iowa US 34 Field Study

February 2016

Sponsored by
Federal Highway Administration (DTFH 61-06-H-00011 (Work Plan #18))
FHWA TPF-5(183): California, Iowa (lead state), Michigan, Pennsylvania, Wisconsin
About the National CP Tech Center

The mission of the National Concrete Pavement Technology (CP Tech) Center is to unite key transportation stakeholders around the central goal of advancing concrete pavement technology through research, tech transfer, and technology implementation.

About CEER

The mission of the Center for Earthworks Engineering Research (CEER) at Iowa State University is to be the nation's premier institution for developing fundamental knowledge of earth mechanics, and creating innovative technologies, sensors, and systems to enable rapid, high quality, environmentally friendly, and economical construction of roadways, aviation runways, railroad embankments, dams, structural foundations, fortifications constructed from earth materials, and related geotechnical applications.

Disclaimer Notice

The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the information presented herein. The opinions, findings and conclusions expressed in this publication are those of the authors and not necessarily those of the sponsors.

The sponsors assume no liability for the contents or use of the information contained in this document. This report does not constitute a standard, specification, or regulation.

The sponsors do not endorse products or manufacturers. Trademarks or manufacturers’ names appear in this report only because they are considered essential to the objective of the document.

Iowa State University Non-Discrimination Statement

Iowa State University does not discriminate on the basis of race, color, age, ethnicity, religion, national origin, pregnancy, sexual orientation, gender identity, genetic information, sex, marital status, disability, or status as a U.S. veteran. Inquiries regarding non-discrimination policies may be directed to Office of Equal Opportunity, Title IX/ADA Coordinator, and Affirmative Action Officer, 3350 Beardshear Hall, Ames, Iowa 50011, 515-294-7612, email eooffice@iastate.edu.

Iowa Department of Transportation Statements

Federal and state laws prohibit employment and/or public accommodation discrimination on the basis of age, color, creed, disability, gender identity, national origin, pregnancy, race, religion, sex, sexual orientation or veteran's status. If you believe you have been discriminated against, please contact the Iowa Civil Rights Commission at 800-457-4416 or the Iowa Department of Transportation affirmative action officer. If you need accommodations because of a disability to access the Iowa Department of Transportation's services, contact the agency's affirmative action officer at 800-262-0003.

The preparation of this report was financed in part through funds provided by the Iowa Department of Transportation through its “Second Revised Agreement for the Management of Research Conducted by Iowa State University for the Iowa Department of Transportation” and its amendments.

The opinions, findings, and conclusions expressed in this publication are those of the authors and not necessarily those of the Iowa Department of Transportation or the U.S. Department of Transportation Federal Highway Administration.
Improving the Foundation Layers for Pavements:
Field Assessment of Jointed Portland Cement Concrete Pavement with Premature Distresses – Iowa US 34 Field Study

February 2016

David J. White, Pavana Vennapusa, Yang Zhang

National Concrete Pavement Technology Center and Center for Earthworks Engineering Research (CEER)
Iowa State University
2711 South Loop Drive, Suite 4700
Ames, IA 50010-8664

Federal Highway Administration
U.S. Department of Transportation
1200 New Jersey Avenue SE
Washington, DC 20590

Visit www.cptechcenter.org or www.ceer.iastate.edu for color PDF files of this and other research reports.

This technical project report is one of the field project reports developed as part of the TPF-5(183) and FHWA DTFH 61-06-H-00011:WO18 studies.

The Iowa Department of Transportation (DOT) identified that a few sections of pavement on US Highway 34 near Mount Pleasant, Iowa showed early deterioration in ride quality. The Iowa State University research team conducted a visual survey of the cracked panels and the natural geography of the area and performed in situ falling weight deflectometer testing near center and joint of 140 panels over a span of about 830 m. Of the 140 panels, 25 panels showed distresses. The surface layer consisted of nominal 260 mm (10 in.) thick jointed portland cement concrete pavement (JPCP) placed over 150 to 260 mm (6 to 10 in.) thick subbase layer. Based on the project drawings, grading in the tested span required fills up to 10 m and cuts about to 3 m. Of the 830 m test span, about 480 m consisted of subgrade constructed with fill materials, about 350 m consisted of natural subgrade constructed in cut.

FWD tests were conducted to evaluate differences in the deflection basin parameters and the modulus of subgrade reaction ($k$) values between the cracked and uncracked panels, and cut and fill areas. Statistical t-test analysis was conducted to compare the measurement values obtained on panels with and without cracks and in cut and fill areas.

All cracked panels were located in the cut areas. Results showed statistically significant differences in the FWD test results between cracked and uncracked panels, with results on uncracked panels representing better support conditions than on cracked panels. Similarly, values in the fill areas showed support conditions that are better than in the cut areas. The $k$ values were on average about 1.3 times lower under cracked panels than under uncracked panels. The COV of the $k$ values were higher under cracked panels (38%) than under the uncracked panels (23%).

All cracked panels were located in the cut areas. Results showed statistically significant differences in the FWD test results between cracked and uncracked panels, with results on uncracked panels representing better support conditions than on cracked panels. Similarly, values in the fill areas showed support conditions that are better than in the cut areas. The $k$ values were on average about 1.3 times lower under cracked panels than under uncracked panels. The COV of the $k$ values were higher under cracked panels (38%) than under the uncracked panels (23%).
IMPROVING THE FOUNDATION LAYERS FOR CONCRETE PAVEMENTS: FIELD ASSESSMENT OF JOINTED PORTLAND CEMENT CONCRETE PAVEMENT WITH PREMATURE DISTRESSES – IOWA US 34 FIELD STUDY

Technical Report
February 2016

Research Team Members
Tom Cackler, David J. White, Jeffrey R. Roesler, Barry Christopher, Andrew Dawson, Heath Gieselman, and Pavana Vennapusa

Report Authors
David J. White, Pavana K. R. Vennapusa,
Yang Zhang
Iowa State University

Sponsored by
the Federal Highway Administration (FHWA)
DTFH61-06-H-00011 Work Plan 18
FHWA Pooled Fund Study TPF-5(183): California, Iowa (lead state), Michigan, Pennsylvania, Wisconsin

Preparation of this report was financed in part through funds provided by the Iowa Department of Transportation through its Research Management Agreement with the Institute for Transportation (InTrans Project 09-352)

National Concrete Pavement Technology Center and Center for Earthworks Engineering Research (CEER)
Iowa State University
2711 South Loop Drive, Suite 4700
Ames, IA 50010-8664
Phone: 515-294-5768
www.cptechcenter.org and www.ceer.iastate.edu
# TABLE OF CONTENTS

ACKNOWLEDGMENTS ............................................................................................................. ix
LIST OF ACRONYMS AND SYMBOLS.................................................................................... xi
EXECUTIVE SUMMARY ......................................................................................................... xiii
CHAPTER 1. INTRODUCTION ....................................................................................................1
CHAPTER 2. EXPERIMENTAL TESTING METHODS ....................................................................2
  Falling Weight Deflectometer ....................................................................................... 2
  Statistical Analysis ........................................................................................................ 7
CHAPTER 3. FIELD TEST RESULTS, OBSERVATIONS, AND ANALYSIS ..............................9
CHAPTER 4. SUMMARY AND CONCLUSIONS ................................................................... 34
REFERENCES ......................................................................................................................... 35
LIST OF FIGURES

Figure 1. FWD deflection sensor setup used for this study and an example deflection basin........2
Figure 2. Void detection using load-deflection data from FWD test........................................3
Figure 3. Static $k_{PLT}$ values versus $k_{FWD-Dynamic}$ measurements reported in literature ..........7
Figure 4. Longitudinal cracking near Sta. 350+00 (7/27/12)......................................................9
Figure 5. Faulting measured along longitudinal crack near Sta. 350+00 (7/27/12) ...................10
Figure 6. Corner cracking observed near Sta. 350+25 (7/27/12).................................................10
Figure 7. Mid panel cracking observed near Sta. 349 (7/27/12)....................................................11
Figure 8. Longitudinal cracking near Sta. 348+50 near mile post 194 (7/27/12) ......................11
Figure 9. Close-up views of the cracks near mile post 194 (7/27/12)...........................................12
Figure 10. Midpanel cracking near on panel 32 near mile post 194 (7/27/12)..............................13
Figure 11. Corner cracking on panel 84 (7/27/12)......................................................................14
Figure 12. Longitudinal and mid-panel cracking on panel 87 (7/27/12)......................................15
Figure 13. Looking down the creek valley near Sta. 347+00 (7/27/12)..................................16
Figure 14. Cracks observed on embankment fill slope near Sta. 347+00 (7/27/12) .................17
Figure 15. FWD $D_0$ versus distance with applied loads of (a) 80 kN (18,000 lb) and (b) 40 kN (9,000 lb) [Distance 0 = Sta 350+50]........................................................................18
Figure 16. Joint LTE from FWD tests versus distance with applied loads of (a) 80 kN (18,000 lb) and (b) 40 kN (9,000 lb) [Distance 0 = Sta 350+50] .......................................................19
Figure 17. I-value versus distance with applied loads of (a) 80 kN (18,000 lb) and (b) 40 kN (9,000 lb) [Distance 0 = Sta 350+50].................................................................20
Figure 18. $k_{FWD-Static-Corr}$ versus distance with applied loads of (a) 80 kN (18,000 lb) and (b) 40 kN (9,000 lb) [Distance 0 = Sta 350+50] .................................................................21
Figure 19. SCI versus distance with applied loads of (a) 80 kN (18,000 lb) and (b) 40 kN (9,000 lb) [Distance 0 = Sta 350+50].................................................................22
Figure 20. BDI versus distance with applied loads of (a) 80 kN (18,000 lb) and (b) 40 kN (9,000 lb) [Distance 0 = Sta 350+50].................................................................23
Figure 21. BCI versus distance with applied loads of (a) 80 kN (18,000 lb) and (b) 40 kN (9,000 lb) [Distance 0 = Sta 350+50].................................................................24
Figure 22. AF versus distance with applied loads of (a) 80 kN (18,000 lb) and (b) 40 kN (9,000 lb) [Distance 0 = Sta 350+50].................................................................25
Figure 23. Box plots of FWD deflection basin parameters near mid-panel comparing panels with and without cracks: (a) $D_0$, (b) BCI, (c) BDI, (d) AF, (e) I-value, (f) $k_{FWD-Static-Corr}$, and (g) SCI ..................................................27
Figure 24. Box plots of FWD deflection basin parameters near joint comparing panels with and without cracks: (a) $D_0$, (b) BCI, (c) BDI, (d) AF, (e) joint LTE, and (f) SCI ..............28
Figure 25. Box plots of FWD deflection basin parameters near mid-panel comparing panels located in fill and cut areas: (a) $D_0$, (b) BCI, (c) BDI, (d) AF, (e) I-value, (f) $k_{FWD-Static-Corr}$, and (g) SCI ..................................................31
Figure 26. Box plots of FWD deflection basin parameters near joint comparing panels located in cut and fill areas: (a) $D_0$, (b) BCI, (c) BDI, (d) AF, (e) joint LTE, and (f) SCI ....................................................................32
LIST OF TABLES

Table 1. Summary of $t$ test analysis results on FWD deflection basin parameters near mid-panel on cracked versus uncracked panels .................................................................29
Table 2. Summary of $t$ test analysis results on FWD deflection basin parameters near joint on uncracked versus cracked panels ..................................................................................29
Table 3. Summary of $t$ test analysis results on FWD deflection basin parameters near mid-panel in cut versus fill areas ...........................................................................................33
Table 4. Summary of $t$ test analysis results on FWD deflection basin parameters near joint in cut versus fill areas ........................................................................................................33
ACKNOWLEDGMENTS

This research was conducted under Federal Highway Administration (FHWA) DTFH61-06-H-00011 Work Plan 18 and the FHWA Pooled Fund Study TPF-5(183), involving the following state departments of transportation:

- California
- Iowa (lead state)
- Michigan
- Pennsylvania
- Wisconsin

The authors would like to express their gratitude to the National Concrete Pavement Technology (CP Tech) Center, the FHWA, the Iowa Department of Transportation (DOT), and the other pooled fund state partners for their financial support and technical assistance.

Jay Ridlen, Douglas Swan, Kenneth Morrow, and several others with the Iowa DOT provided background information on the project and access the field sections. Dustin Wheatley with Iowa State University provided assistance with field testing. Christianna White provided assistance with technical editing. We greatly appreciate their help.
LIST OF ACRONYMS AND SYMBOLS

<table>
<thead>
<tr>
<th>Acronym</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>BCI</td>
<td>Base curvature index</td>
</tr>
<tr>
<td>BDI</td>
<td>Base damage index</td>
</tr>
<tr>
<td>COV</td>
<td>Coefficient of variation</td>
</tr>
<tr>
<td>$D_0$</td>
<td>Deflection measured under the plate</td>
</tr>
<tr>
<td>$D_0^*$</td>
<td>Non-dimensional deflection coefficient</td>
</tr>
<tr>
<td>$D_1$ to $D_7$</td>
<td>Deflections measured away from the plate at various set distances</td>
</tr>
<tr>
<td>FWD</td>
<td>Falling weight deflectometer</td>
</tr>
<tr>
<td>I</td>
<td>Intercept</td>
</tr>
<tr>
<td>IRI</td>
<td>International roughness index</td>
</tr>
<tr>
<td>$k$</td>
<td>Modulus of subgrade reaction</td>
</tr>
<tr>
<td>$k_{FWD-Dynamic}$</td>
<td>Dynamic modulus of subgrade reaction from FWD test</td>
</tr>
<tr>
<td>$k_{FWD-Static}$</td>
<td>Static modulus of subgrade reaction from FWD test</td>
</tr>
<tr>
<td>$k_{FWD-Static-Corr}$</td>
<td>Static modulus of subgrade reaction from FWD test corrected for finite slab size</td>
</tr>
<tr>
<td>L</td>
<td>Relative stiffness</td>
</tr>
<tr>
<td>LTE</td>
<td>Load transfer efficiency</td>
</tr>
<tr>
<td>n</td>
<td>Number of measurements</td>
</tr>
<tr>
<td>P</td>
<td>Applied load</td>
</tr>
<tr>
<td>PCC</td>
<td>Portland cement concrete</td>
</tr>
<tr>
<td>r</td>
<td>Plate radius</td>
</tr>
<tr>
<td>SCI, BCI, BDI, AF</td>
<td>FWD deflection basin parameters</td>
</tr>
<tr>
<td>$\mu$</td>
<td>Statistical mean or average</td>
</tr>
<tr>
<td>$\sigma$</td>
<td>Statistical standard deviation</td>
</tr>
</tbody>
</table>
EXECUTIVE SUMMARY

Quality foundation layers (the natural subgrade, subbase, and embankment) are essential to achieving excellent pavement performance. Unfortunately, many pavements in the United States still fail due to inadequate foundation layers. To address this problem, a research project, Improving the Foundation Layers for Pavements (FHWA DTFH 61-06-H-00011 WO #18; FHWA TPF-5(183)), was undertaken by Iowa State University (ISU) to identify, and provide guidance for implementing, best practices regarding foundation layer construction methods, material selection, in situ testing and evaluation, and performance-related designs and specifications. As part of the project, field studies were conducted in several in-service concrete pavements across the country that represented either premature failures or successful long-term pavements. A key aspect of each field study was to tie performance of the foundation layers to key engineering properties and pavement performance. In situ foundation layer performance data, as well as original construction data and maintenance/rehabilitation history data, were collected and geospatially and statistically analyzed to determine the effects of site-specific foundation layer construction methods, site evaluation, materials selection, design, treatments, and maintenance procedures on the performance of the foundation layers and of the related pavements. A technical report was prepared for each field study.

The Iowa Department of Transportation (DOT) identified that a few sections of pavement on US Highway 34 near Mount Pleasant, Iowa showed early deterioration in ride quality due to faulting, settlement, and longitudinal/transverse cracking. The identified sections were located between mile posts 208 and 200 and 194.5 to 193.7 on the west bound (WB) lane of US34. The section between mile posts 194.5 and 193.7 was selected for field testing in this study.

The Iowa State University (ISU) research team visited the site on June 25, 2012 and conducted a visual survey of the cracked panels and the natural geography of the area and performed in situ falling weight deflectometer testing near center and joint of 140 panels over a span of about 830 m. The start location of the testing was at Sta. 350+50. Of the 140 panels, 25 panels showed distresses ranging from longitudinal and transverse cracking, mid-panel cracking, corner cracking, and faulting. The ISU research team reviewed the as-built plans and cross-sections of the project site. The surface layer consisted of nominal 260 mm (10 in.) thick jointed portland cement concrete pavement (JPCP) placed over 150 to 260 mm (6 to 10 in.) thick subbase layer. Based on the project drawings, grading in the tested span required fills up to 10 m and cuts about to 3 m. Of the 830 m test span, about 300 m consisted of subgrade constructed with fill materials, about 530 m consisted of natural subgrade constructed in cut.

This report presents the field observations of the ISU research team and results and analysis of in situ falling weight deflectometer tests conducted on US34 WB between mile posts 194.5 and 196.7. FWD tests were conducted to evaluate differences in the deflection basin parameters and the modulus of subgrade reaction (k) values between the cracked and uncracked panels, and cut and fill areas. Statistical t-test analysis was conducted to compare the measurement values obtained on panels with and without cracks and in cut and fill areas. Pictures documenting the distresses observed on the pavement surface and cracks observed on embankment fill slopes are presented in this report.
Following are the key findings from this study:

- All of the cracked panels were located in the cut areas. Distresses observed on the pavement surface included longitudinal cracks, transverse cracks, mid-panel cracks, corner cracks, and faulting.
- Tension cracks were observed on the slope where about 10 m thick embankment fill was placed, which suggest possibility of slope movements.
- The $D_0$, $k_{FWD-Static-Corr}$, SCI, BDI, and BCI values showed statistically significant differences between cracked and uncracked panels, with results on the uncracked panels representing better support conditions than on the cracked panels.
- The $D_0$, $k_{FWD-Static-Corr}$, SCI, BDI, and BCI values showed statistically significant differences between cut and fill areas, with results in the fill areas showing better support conditions than in the cut areas. (Note that all cracked panels were located in the cut area).
- The $k_{FWD-Static-Corr}$ values were on average about 1.3 times lower under cracked panels than under uncracked panels. The COV of the $k$ values were higher under the cracked panels (38%) than under the uncracked panels (23%).
- The $k_{FWD-Static-Corr}$ values were on average about 1.1 times lower in cut areas than in fill areas. The COV of the $k$ values were higher in the cut areas (31%) than in the fill areas (21%).
- There was no statistically significant difference in the I values between the cracked and uncracked panels and the cut and fill areas. The I values were all very low ($\leq 1 \mu m$). $I > 5 \mu m$ is typically considered a trigger value suggesting void beneath the pavement.
- The joint LTE at all panels was relatively high ($> 91\%$), and there was no statistically significant difference between the cracked and the uncracked panels and the cut and fill areas.
CHAPTER 1. INTRODUCTION

The Iowa Department of Transportation (DOT) identified that a few sections of pavement on US Highway 34 near Mount Pleasant, Iowa showed early deterioration in ride quality due to faulting, settlement, and longitudinal/transverse cracking. The identified sections were located between mile posts 208 and 200 and 194.5 to 193.7 on the west bound (WB) lane of US34. The section between mile posts 194.5 and 193.7 was selected for field testing in this study.

The Iowa State University (ISU) research team visited the site on June 25, 2012 and conducted a visual survey of the cracked panels and the natural geography of the area and performed in situ falling weight deflectometer testing near center and joint of 140 panels over a span of about 830 m. The start location of the testing was at Sta. 350+50. Of the 140 panels, 25 panels showed distresses ranging from longitudinal and transverse cracking, mid-panel cracking, corner cracking, and faulting. Some of the cracked panels were patched with asphalt at the time of testing. All tests were conducted on the outside (right) lane. Traffic closure during testing was provided by Iowa DOT personnel.

The ISU research team reviewed the as-built plans and cross-sections of the project site. The surface layer consisted of nominal 260 mm (10 in.) thick jointed portland cement concrete pavement (JPCP) placed over 150 to 260 mm (6 to 10 in.) thick subbase layer. Based on the project drawings, grading in the tested span required fills up to 10 m and cuts about to 3 m. Of the 830 m test span, about 300 m consisted of subgrade constructed with fill materials and about 530 m consisted of natural subgrade constructed in cut.

This report presents the field observations of the ISU research team and results and analysis of in situ falling weight deflectometer tests. The FWD tests were conducted to evaluate differences in the deflection basin parameters and the modulus of subgrade reaction (k) values between the cracked and uncracked panels, and cut and fill areas.

Chapter 2 describes the FWD test procedure, the parameters calculated from the FWD data, and the statistical analysis procedures used in this study. Chapter 3 presents the results and analysis. Chapter 4 presents the key findings from this study.
CHAPTER 2. EXPERIMENTAL TESTING METHODS

Falling Weight Deflectometer

Falling weight deflectometer (FWD) tests were conducted using a Kuab FWD setup with a 300 mm (11.81 in) diameter loading plate by applying one seating drop and three loading drops. The applied loads varied from about 27 kN (6,000 lb) to 54 kN (12,000 lb) in the three loading drops. The actual applied loads were recorded using a load cell, and deflections were recorded using seismometers mounted on the device, per ASTM D4694-09 Standard Test Method for Deflections with a Falling-Weight-Type Impulse Load Device. The FWD plate and deflection sensor setup and a typical deflection basin are shown in Figure 1. To compare deflection values from different test locations at the same applied contact stress, the values at each test location were normalized to a 40 kN (9,000 lb) applied force.

![FWD plate and deflection sensors setup (Plan View)](image1)

![Deflection Basin](image2)

Figure 1. FWD deflection sensor setup used for this study and an example deflection basin

FWD tests were near mid panel and at joints. Tests conducted at the joints were used to determine joint load transfer efficiency (LTE) and voids beneath the pavement based on “zero”
load intercept values. Tests conducted at the center of the slab panels were used to determine modulus of subgrade reaction \((k)\) values and the intercept values. The procedures used to calculate these parameters are described below.

LTE was determined by obtaining deflections under the plate on the loaded slab \((D_0)\) and deflections of the unloaded slab \((D_1)\) using a sensor positioned about 305 mm (12 in.) away from the center of the plate (Figure 1). The LTE was calculated using Equation 4.

\[
LTE(\%) = \frac{D_1}{D_0} \times 100
\]

Voids underneath pavements can be detected by plotting the applied load measurements on the X-axis and the corresponding deflection measurements on the y-axis and plotting a best fit linear regression line, as illustrated in Figure 2, to determine the “zero” load intercept \((I)\) values. AASHTO (1993) suggests \(I = 0.05\) mm (2 mils) as a critical value for void detection. According to von Quintus and Simpson (2002), if \(I = -0.01\) and \(+0.01\) mm, then the response would be considered elastic. If \(I > 0.01\) then the response would be considered deflection hardening, and if \(I < -0.01\) then the response would be considered deflection softening.

![Figure 2. Void detection using load-deflection data from FWD test](image)

Pavement layer temperatures at different depths were obtained during FWD testing, in accordance with the guidelines from Schmalzer (2006). The temperature measurements were used to determine equivalent linear temperature gradients \((T_l)\) following the temperature-moment concept suggested by Janssen and Snyder (2000). According to Vandenbossche (2005), \(I\)-values are sensitive to temperature induced curling and warping affects. Large positive temperature gradients (i.e., when the surface is warmer than the bottom) that cause the panel corners to curl down result in false negative \(I\)-values. Conversely, large negative gradients (i.e., when the surface is cooler than the bottom) that cause the panel corners to curl upward result in
false positive I-values. Interpretation of I-values therefore should consider the temperature gradient. Concerning LTE measurements for doweled joints, the temperature gradient is reportedly not a critical factor (Vandenbossche 2005).

The SCI, BDI, BCI, and AF measurements are referred to as deflection basin parameters and are determined using the following equations:

\[
SCI \ (\text{mm}) = D_0 - D_2
\]

\[
BDI \ (\text{mm}) = D_2 - D_4
\]

\[
BCI \ (\text{mm}) = D_4 - D_5
\]

\[
AF \ (\text{mm}) = \frac{152.4 \times (D_0 + 2D_2 + 2D_4 + D_5)}{D_0}
\]

where, \(D_0\) = peak deflection measured directly beneath the plate, \(D_2\) = peak deflection measured at 305 mm away from the plate center, \(D_4\) = peak deflection measured at 510 mm away from the plate centre, and \(D_5\) = peak deflection measured at 914 mm away from the plate centre.

According to Horak (1987), the SCI parameter provides a measure of the strength/ stiffness of the upper portion (base layers) of the pavement foundation layers (Horak 1987). Similarly, BDI represents layers between 300 mm and 600 mm depth (base and subbase layers) and BCI represents layers between 600 mm and 900 mm depth (subgrade layers) from the surface (Kilareski and Anani 1982). The AF is primarily the normalized (with \(D_0\)) area under the deflection basin curve up to sensor \(D_5\) (AASHTO 1993). AF has been used to characterize variations in the foundation layer material properties by some researchers (e.g., Stubstad 2002). Comparatively, lower SCI or BDI or BCI or AF values indicate better support conditions (Horak 1987).

The \(k\) values were determined using the AREA\(_4\) method described in AASHTO (1993). Since the \(k\) value determined from FWD test represents a dynamic value, it is referred to here as \(k_{\text{FWD-Dynamic}}\). Deflections obtained from four sensors (\(D_0, D_2, D_4,\) and \(D_5\) shown in Figure 1 were used in the AREA\(_4\) calculation. The AREA method was first proposed by Hoffman and Thompson (1981) for flexible pavements and has since been applied extensively for concrete pavements (Darter et al. 1995). AREA\(_4\) is calculated using Equation 5 and has dimensions of length (in inches), as it is normalized with deflections under the center of the plate (\(D_0\)):

\[
AREA_4 = 6 + 12 \times \left( \frac{D_2}{D_0} \right) + 12 \times \left( \frac{D_4}{D_0} \right) + 6 \times \left( \frac{D_5}{D_0} \right)
\]
where $D_0 = \text{deflections measured directly under the plate (in.)}$; $D_2 = \text{deflections measured at 305 mm (12 in.) away from the plate center (in.)}$; $D_4 = \text{deflections measured at 610 mm (24 in.) away from the plate center (in.)}$; and $D_5 = \text{deflections measured at 914 mm (36 in.) away from the plate center (in.)}$. The AREA$_4$ method can also be calculated using different sensor configurations and setups, (i.e., using deflection data from 3, 5, or 7 sensors), and those methods are described in detail in the literature (Substad et al. 2006, Smith et al. 2007).

In early research conducted using the AREA method, the ILLI-SLAB finite element program was used to compute a matrix of maximum deflections at the plate center and the AREA values by varying the subgrade $k$, the modulus of the PCC layer, and the thickness of the slab (ERES Consultants, Inc. 1982). Measurements obtained from FWD tests were then compared with the ILLI-SLAB program results to determine the $k$ values through back calculation. Barenberg and Petros (1991) and Ioannides (1990) proposed a forward solution procedure based on Westergaard’s solution for loading on an infinite plate to replace the back calculation procedure. This forward solution presented a unique relationship between AREA value (for a given load and sensor arrangement) and the dense liquid radius of relative stiffness $(L)$ in which subgrade is characterized by the $k$ value. The radius of relative stiffness $(L)$ is estimated using Equation 6:

$$L = \left[ \frac{\ln \left( \frac{x_1 - \text{AREA}_4}{x_2} \right)}{x_3} \right]^{x_4} \tag{6}$$

where $x_1 = 36$, $x_2 = 1812.279$, $x_3 = -2.559$, $x_4 = 4.387$. It must be noted that the $x_1$ to $x_4$ values vary with the sensor arrangement and these values are only valid for the AREA$_4$ sensor setup. Once, the $L$ value is known, the $k_{\text{FWD-Dynamic}}$ value can be estimated using Equation 7:

$$k_{\text{FWD-Dynamic}} (pci) = \frac{PD_0^*}{D_0L^2} \tag{7}$$

where $P = \text{applied load (lbs)}$, $D_0 = \text{deflection measured at plate center (inches)}$, and $D_0^* = \text{non-dimensional deflection coefficient calculated using Equation 8:}$

$$D_0^* = a \cdot e^{-bc^{-L}} \tag{8}$$

where $a = 0.12450$, $b = 0.14707$, $c = 0.07565$. It must be noted that these equations and coefficients are valid for an FWD setup with an 11.81 in. diameter plate.

The advantages of the AREA$_4$ method are the ease of use without back calculations and the use of multiple sensor data. The disadvantages are that the process assumes that the slab and the subgrade are horizontally infinite. This assumption leads to underestimating the $k$ values of...
jointed pavements. Crovetti (1993) developed the following slab size corrections for a square slab that is based on finite element analysis conducted using the ILLI-SLAB program and is for use in the $k_{\text{FWD-Dynamic}}$:

$$Adjusted \ D_0 = D_0 \left( 1 - 1.15085 e^{-0.71878 \left( \frac{L'}{L} \right)^{0.80151}} \right)$$

$$Adjusted \ L = L \left( 1 - 0.89434 e^{-0.61662 \left( \frac{L'}{L} \right)^{1.04831}} \right)$$

where $L'$ = slab size (smaller dimension of a rectangular slab, length or width). This procedure also has limitations: (1) it considers only a single slab with no load transfer to adjacent slabs, and (2) it assumes a square slab. The square slab assumption is considered to produce sufficiently accurate results when the smaller dimension of a rectangular slab is assumed as $L'$ (Darter et al. 1995). Darter et al. (1995) suggested using $L' = \sqrt{\text{Length} \times \text{Width}}$ to further refine slab size corrections. However, no established procedures for correcting for load transfer to adjacent slabs have been reported, so accounting for load transfer remains as a limitation of this method.

AASHTO (1993) suggests dividing the $k_{\text{FWD-Dynamic}}$ value by a factor of 2 to determine the equivalent $k_{\text{FWD-Static}}$ value. The origin of this factor 2 dates back to Foxworthy’s work in the 1980s. Foxworthy (1985) reported comparisons between the $k_{\text{FWD-Dynamic}}$ values obtained using Dynatest model 8000 FWD and the Static $k$ values (Static $k_{\text{PLT}}$) obtained from 30 in. diameter plate load tests (the exact procedure followed to calculate the Static $k_{\text{PLT}}$ is not reported in Foxworthy 1985). Foxworthy used the AREA based back calculation procedure using the ILLI-SLAB finite element program. Results obtained from Foxworthy’s study (Figure 3) are based on 7 FWD tests conducted on PCC pavements with slab thicknesses varying from about 10 in. to 25.5 in. and plate load tests conducted on the foundation layer immediately beneath the pavement over a 4 ft x 5 ft test area. A few of these sections consisted of a 5 to 12 in. thick base course layer and some did not. The subgrade layer material consisted of CL soil from Sheppard Air Force Base in Texas, SM soil from Seymour-Johnson Air Force Base in North Carolina, and an unspecified soil type from McDill Air Force base in Florida. No slab size correction was performed on this dataset.

Data from Foxworthy (1985) yielded a logarithmic relationship between the dynamic and the static $k$ values. On average, the $k_{\text{FWD-Dynamic}}$ values were about 2.4 times greater than the Static $k_{\text{PLT}}$ values. Darter et al. (1995) indicated that the factor 2 is reasonable based on results from other test sites (Figure 3). Darter et al. (1995) also compared FWD test data from eight long-term pavement performance (LTPP) test sections with the Static $k_{\text{PLT}}$ values and reported factors ranging from 1.78 to 2.16, with an average of about 1.91. The $k_{\text{FWD-Dynamic}}$ values used in that comparison were corrected for slab size. For the analysis conducted in this research project, the corrected $k_{\text{FWD-Dynamic}}$ values (for finite slab size) were divided by 2 and are reported as $k_{\text{FWD-Static-Corr}}$ values.
Figure 3. Static $k_{PLT}$ values versus $k_{FWD}$-Dynamic measurements reported in literature

Statistical Analysis

Student $t$-test analysis (Ott and Longnecker 2008) was conducted to assess differences between results obtained on cracked and uncracked panels, using the following equations:

\[
\frac{t}{s_p} = \frac{\mu_0 - \mu_1}{\sqrt{\frac{1}{n_0} + \frac{1}{n_1}}} 
\]

where,

\[
s_p = \sqrt{\frac{(n_0-1)s_0^2 + (n_1-1)s_1^2}{n_0 + n_1 - 2}}
\]

$n_0$ and $n_1$ = number of measurements obtained on cracked or uncracked section, respectively; $s_p$ = pooled standard deviation; and $s_0$ and $s_1$ = standard deviation of measurements obtained on cracked or uncracked sections, respectively.
The observed $t$-values were compared with the minimum $t$-value for a one-tailed test with degree of freedom (df) = $n_0 + n_1 - 2$, for 95% confidence level (i.e., $\alpha = 0.05$). When comparing measurements from cracked or uncracked sections, if the $t$-values were greater than the minimum $t$-value, then it was concluded that there is sufficient evidence that the measurements were statistically different.
CHAPTER 3. FIELD TEST RESULTS, OBSERVATIONS, AND ANALYSIS

Pictures in Figure 4 to Figure 12 show the various distresses observed on the pavement surface layer, such as longitudinal cracks, transverse cracks, mid-panel cracks, corner cracks, and faulting.

Figure 13 and Figure 14 show pictures of embankment slope on the north side of US34 near Sta. 347+00 that was built up with about 10 m thick embankment fill material. Tension cracks were observed on the slope as shown in Figure 14, which suggests the possibility of slope movements.

FWD test results measurements obtained near joints and mid-panel are presented in Figure 15 to Figure 22. The figures identify zones of cracked panels and cut or fill. All of the cracked panels are located in the cut areas.

Figure 4. Longitudinal cracking near Sta. 350+00 (7/27/12)
Figure 5. Faulting measured along longitudinal crack near Sta. 350+00 (7/27/12)

Figure 6. Corner cracking observed near Sta. 350+25 (7/27/12)
Figure 7. Mid panel cracking observed near Sta. 349 (7/27/12)

Figure 8. Longitudinal cracking near Sta. 348+50 near mile post 194 (7/27/12)
Figure 9. Close-up views of the cracks near mile post 194 (7/27/12)
Figure 10. Midpanel cracking near on panel 32 near mile post 194 (7/27/12)
Figure 11. Corner cracking on panel 84 (7/27/12)
Figure 12. Longitudinal and mid-panel cracking on panel 87 (7/27/12)
Figure 13. Looking down the creek valley near Sta. 347+00 (7/27/12).
Figure 14. Cracks observed on embankment fill slope near Sta. 347+00 (7/27/12)
Figure 15. FWD $D_0$ versus distance with applied loads of (a) 80 kN (18,000 lb) and (b) 40 kN (9,000 lb) [Distance 0 = Sta 350+50]
Figure 16. Joint LTE from FWD tests versus distance with applied loads of (a) 80 kN (18,000 lb) and (b) 40 kN (9,000 lb)

[Distance 0 = Sta 350+50]
Figure 17. I-value versus distance with applied loads of (a) 80 kN (18,000 lb) and (b) 40 kN (9,000 lb)

[Distance 0 = Sta 350+50]
Figure 18. kFWD-Static-Corr versus distance with applied loads of (a) 80 kN (18,000 lb) and (b) 40 kN (9,000 lb) [Distance 0 = Sta 350+50]
Figure 19. SCI versus distance with applied loads of (a) 80 kN (18,000 lb) and (b) 40 kN (9,000 lb) [Distance 0 = Sta 350+50]
Figure 20. BDI versus distance with applied loads of (a) 80 kN (18,000 lb) and (b) 40 kN (9,000 lb) [Distance 0 = Sta 350+50]
Figure 21. BCI versus distance with applied loads of (a) 80 kN (18,000 lb) and (b) 40 kN (9,000 lb) [Distance 0 = Sta 350+50]
Figure 22. AF versus distance with applied loads of (a) 80 kN (18,000 lb) and (b) 40 kN (9,000 lb) [Distance 0 = Sta 350+50]
Box plots showing FWD test measurements obtained on panels with and without cracks are shown along with number of measurements (n), mean, and standard deviation statistics in Figure 23 and Figure 24 for tests conducted near mid-panel and joint, respectively. Summaries of t-test analysis results that compare measurement values obtained on panels with and without cracks are provided in Table 1 for tests conducted near mid-panel and Table 2 for tests near joint.

Following are the key findings from the statistical analysis test results:

- The $D_0$, $k_{FWD-Static-Corr}$, SCI, BDI, and BCI values showed statistically significant differences between cracked and uncracked panels, with results on uncracked panels representing better support conditions than on cracked panels.

- The $k_{FWD-Static-Corr}$ values were on average about 1.3 times lower under cracked panels than under uncracked panels. The COV of the $k$ values were higher under cracked panels (38%) than under the uncracked panels (23%).

- There was no statistically significant difference in the I values between the cracked and uncracked panels. The I values were all very low ($\leq 1$ µm). $I > 5$ µm is typically considered a trigger value suggesting void beneath the pavement.

- The joint LTE at all panels was relatively high (> 91%) and there was no statistically significant difference between the cracked and the uncracked panels.
Figure 23. Box plots of FWD deflection basin parameters near mid-panel comparing panels with and without cracks: (a) D0, (b) BCI, (c) BDI, (d) AF, (e) I-value, (f) $k_{FWD-Static-Corr}$, and (g) SCI
Figure 24. Box plots of FWD deflection basin parameters near joint comparing panels with and without cracks: (a) D0, (b) BCI, (c) BDI, (d) AF, (e) joint LTE, and (f) SCI
Table 1. Summary of t test analysis results on FWD deflection basin parameters near mid-panel on cracked versus uncracked panels

<table>
<thead>
<tr>
<th>Parameter</th>
<th>No crack or crack</th>
<th>Mean</th>
<th>COV (%)</th>
<th>t-value</th>
<th>Pr</th>
</tr>
</thead>
<tbody>
<tr>
<td>D0 (µm)</td>
<td>No Crack</td>
<td>99</td>
<td>20</td>
<td>-4.07</td>
<td>&lt; 0.001</td>
</tr>
<tr>
<td></td>
<td>Crack</td>
<td>135</td>
<td>32</td>
<td></td>
<td></td>
</tr>
<tr>
<td>I (µm)</td>
<td>No Crack</td>
<td>&lt; 1</td>
<td>760</td>
<td>-8.1</td>
<td>0.212</td>
</tr>
<tr>
<td></td>
<td>Crack</td>
<td>1</td>
<td>473</td>
<td></td>
<td></td>
</tr>
<tr>
<td>kFWD-Static-Corr (kPa/mm)</td>
<td>No Crack</td>
<td>29</td>
<td>23</td>
<td>4.06</td>
<td>&lt; 0.001</td>
</tr>
<tr>
<td></td>
<td>Crack</td>
<td>22</td>
<td>38</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SCI (µm)</td>
<td>No Crack</td>
<td>6</td>
<td>23</td>
<td>-2.82</td>
<td>0.005</td>
</tr>
<tr>
<td></td>
<td>Crack</td>
<td>8</td>
<td>41</td>
<td></td>
<td></td>
</tr>
<tr>
<td>BDI (µm)</td>
<td>No Crack</td>
<td>9</td>
<td>14</td>
<td>-3.71</td>
<td>&lt; 0.001</td>
</tr>
<tr>
<td></td>
<td>Crack</td>
<td>12</td>
<td>31</td>
<td></td>
<td></td>
</tr>
<tr>
<td>BCI (µm)</td>
<td>No Crack</td>
<td>9</td>
<td>109</td>
<td>-1.99</td>
<td>0.025</td>
</tr>
<tr>
<td></td>
<td>Crack</td>
<td>11</td>
<td>41</td>
<td></td>
<td></td>
</tr>
<tr>
<td>AF (mm)</td>
<td>No Crack</td>
<td>808</td>
<td>2</td>
<td>-0.93</td>
<td>0.18</td>
</tr>
<tr>
<td></td>
<td>Crack</td>
<td>812</td>
<td>3</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Note: Highlighted cell indicates statistically significant difference at 95% confidence level between the cracked and the uncracked panels.

Table 2. Summary of t test analysis results on FWD deflection basin parameters near joint on uncracked versus cracked panels

<table>
<thead>
<tr>
<th>Parameter</th>
<th>No crack or crack</th>
<th>Mean</th>
<th>COV (%)</th>
<th>t-value</th>
<th>Pr</th>
</tr>
</thead>
<tbody>
<tr>
<td>D0 (µm)</td>
<td>No Crack</td>
<td>106</td>
<td>20</td>
<td>-3.73</td>
<td>&lt; 0.001</td>
</tr>
<tr>
<td></td>
<td>Crack</td>
<td>147</td>
<td>35</td>
<td></td>
<td></td>
</tr>
<tr>
<td>LTE (%)</td>
<td>No Crack</td>
<td>96</td>
<td>2</td>
<td>-0.11</td>
<td>0.46</td>
</tr>
<tr>
<td></td>
<td>Crack</td>
<td>96</td>
<td>3</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SCI (µm)</td>
<td>No Crack</td>
<td>10</td>
<td>32</td>
<td>-2.73</td>
<td>0.006</td>
</tr>
<tr>
<td></td>
<td>Crack</td>
<td>13</td>
<td>36</td>
<td></td>
<td></td>
</tr>
<tr>
<td>BDI (µm)</td>
<td>No Crack</td>
<td>12</td>
<td>22</td>
<td>-3.54</td>
<td>&lt; 0.001</td>
</tr>
<tr>
<td></td>
<td>Crack</td>
<td>16</td>
<td>32</td>
<td></td>
<td></td>
</tr>
<tr>
<td>BCI (µm)</td>
<td>No Crack</td>
<td>10</td>
<td>24</td>
<td>-3.88</td>
<td>&lt; 0.001</td>
</tr>
<tr>
<td></td>
<td>Crack</td>
<td>13</td>
<td>32</td>
<td></td>
<td></td>
</tr>
<tr>
<td>AF (mm)</td>
<td>No Crack</td>
<td>777</td>
<td>2</td>
<td>-1.35</td>
<td>0.094</td>
</tr>
<tr>
<td></td>
<td>Crack</td>
<td>783</td>
<td>3</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Note: Highlighted cell indicates statistically significant difference at 95% confidence level between the cracked and the uncracked panels.

Box plots showing FWD test measurements obtained in cut and fill areas are shown along with number of measurements (n), mean, and standard deviation statistics in Figure 25 and Figure 26 for tests conducted near mid-panel and joint, respectively. Summary of t-test analysis results comparing measurement values obtained on panels in cut and fill areas as provided in Table 3 and Table 4, respectively.

Following are the key findings from the statistical analysis test results:
• The $D_0$, $k_{FWD-Static-Corr}$, SCI, BDI, and BCI values showed statistically significant differences between cut and fill areas, with results in fill areas showing better support conditions than in cut areas. As indicated earlier, all cracked panels were located in the cut areas.

• The $k_{FWD-Static-Corr}$ values were on average about 1.1 times lower in cut areas than in fill areas. The COV of the $k$ values were higher in the cut areas (31%) than in the fill areas (21%).

• There was no statistically significant difference in the I values cut and fill areas. The I values were all very low ($\leq 1 \mu m$).

• The joint LTE at all panels was relatively high (> 91%) and there was no statistically significant difference between tests conducted in cut and fill areas.
Figure 25. Box plots of FWD deflection basin parameters near mid-panel comparing panels located in fill and cut areas: (a) D0, (b) BCI, (c) BDI, (d) AF, (e) I-value, (f) $k_{\text{FWD-Static-Corr}}$, and (g) SCI
Figure 26. Box plots of FWD deflection basin parameters near joint comparing panels located in cut and fill areas: (a) D0, (b) BCI, (c) BDI, (d) AF, (e) joint LTE, and (f) SCI
Table 3. Summary of *t* test analysis results on FWD deflection basin parameters near mid-panel in cut versus fill areas

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Fill or Cut</th>
<th>Mean</th>
<th>COV (%)</th>
<th>t-value</th>
<th>P&lt;sub&gt;r&lt;/sub&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td>D&lt;sub&gt;0&lt;/sub&gt; (µm)</td>
<td>Fill</td>
<td>101</td>
<td>25</td>
<td>-1.77</td>
<td>0.039</td>
</tr>
<tr>
<td></td>
<td>Cut</td>
<td>109</td>
<td>28</td>
<td></td>
<td></td>
</tr>
<tr>
<td>I (µm)</td>
<td>Fill</td>
<td>0</td>
<td>791</td>
<td>-0.47</td>
<td>0.319</td>
</tr>
<tr>
<td></td>
<td>Cut</td>
<td>1</td>
<td>644</td>
<td></td>
<td></td>
</tr>
<tr>
<td>k&lt;sub&gt;FWD-Static-Corr&lt;/sub&gt; (kPa/mm)</td>
<td>Fill</td>
<td>29</td>
<td>21</td>
<td>2.04</td>
<td>0.022</td>
</tr>
<tr>
<td></td>
<td>Cut</td>
<td>27</td>
<td>31</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SCI (µm)</td>
<td>Fill</td>
<td>6</td>
<td>30</td>
<td>-2.46</td>
<td>0.008</td>
</tr>
<tr>
<td></td>
<td>Cut</td>
<td>7</td>
<td>32</td>
<td></td>
<td></td>
</tr>
<tr>
<td>BDI (µm)</td>
<td>Fill</td>
<td>9</td>
<td>14</td>
<td>-2.15</td>
<td>0.017</td>
</tr>
<tr>
<td></td>
<td>Cut</td>
<td>10</td>
<td>26</td>
<td></td>
<td></td>
</tr>
<tr>
<td>BCI (µm)</td>
<td>Fill</td>
<td>10</td>
<td>142</td>
<td>0.342</td>
<td>0.367</td>
</tr>
<tr>
<td></td>
<td>Cut</td>
<td>9</td>
<td>34</td>
<td></td>
<td></td>
</tr>
<tr>
<td>AF (mm)</td>
<td>Fill</td>
<td>808</td>
<td>2</td>
<td>-0.42</td>
<td>0.337</td>
</tr>
<tr>
<td></td>
<td>Cut</td>
<td>809</td>
<td>2</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Note: Highlighted cell indicates statistically significant difference at 95% confidence level between the cut and fill areas

Table 4. Summary of *t* test analysis results on FWD deflection basin parameters near joint in cut versus fill areas

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Cut or Fill</th>
<th>Mean</th>
<th>COV (%)</th>
<th>t-value</th>
<th>P&lt;sub&gt;r&lt;/sub&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td>D&lt;sub&gt;0&lt;/sub&gt; (µm)</td>
<td>Fill</td>
<td>105</td>
<td>21</td>
<td>-2.78</td>
<td>0.003</td>
</tr>
<tr>
<td></td>
<td>Cut</td>
<td>118</td>
<td>31</td>
<td></td>
<td></td>
</tr>
<tr>
<td>LTE (%)</td>
<td>Fill</td>
<td>96</td>
<td>2</td>
<td>-0.08</td>
<td>0.467</td>
</tr>
<tr>
<td></td>
<td>Cut</td>
<td>96</td>
<td>3</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SCI (µm)</td>
<td>Fill</td>
<td>10</td>
<td>36</td>
<td>-2.35</td>
<td>0.010</td>
</tr>
<tr>
<td></td>
<td>Cut</td>
<td>11</td>
<td>33</td>
<td></td>
<td></td>
</tr>
<tr>
<td>BDI (µm)</td>
<td>Fill</td>
<td>12</td>
<td>25</td>
<td>-2.42</td>
<td>0.008</td>
</tr>
<tr>
<td></td>
<td>Cut</td>
<td>13</td>
<td>28</td>
<td></td>
<td></td>
</tr>
<tr>
<td>BCI (µm)</td>
<td>Fill</td>
<td>9</td>
<td>24</td>
<td>-3.07</td>
<td>0.001</td>
</tr>
<tr>
<td></td>
<td>Cut</td>
<td>11</td>
<td>31</td>
<td></td>
<td></td>
</tr>
<tr>
<td>AF (mm)</td>
<td>Fill</td>
<td>780</td>
<td>2</td>
<td>0.92</td>
<td>0.180</td>
</tr>
<tr>
<td></td>
<td>Cut</td>
<td>777</td>
<td>2</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Note: Highlighted cell indicates statistically significant difference at 95% confidence level between the cut and fill areas
CHAPTER 4. SUMMARY AND CONCLUSIONS

This report presented the field observations of the ISU research team and results and analysis of in situ falling weight deflectometer tests conducted on US34 WB between mile posts 194.5 and 196.7. FWD tests were conducted to evaluate differences in the deflection basin parameters and the modulus of subgrade reaction ($k$) values between the cracked and uncracked panels, and cut and fill areas. Statistical t-test analysis was conducted to compare the measurement values obtained on panels with and without cracks and in cut and fill areas. Pictures documenting the distresses observed on the pavement surface and cracks observed on embankment fill slopes are presented in this report.

Following are the key findings from this study:

- All of the cracked panels were located in the cut areas. Distresses observed on the pavement surface included longitudinal cracks, transverse cracks, mid-panel cracks, corner cracks, and faulting.
- Tension cracks were observed on the slope where about 10 m thick embankment fill was placed, which suggest possibility of slope movements.
- The $D_0$, $k_{FWD-Static-Corr}$, SCI, BDI, and BCI values showed statistically significant differences between cracked and uncracked panels, with results on the uncracked panels representing better support conditions than on the cracked panels.
- The $D_0$, $k_{FWD-Static-Corr}$, SCI, BDI, and BCI values showed statistically significant differences between cut and fill areas, with results in the fill areas showing better support conditions than in the cut areas. (Note that all cracked panels were located in the cut area).
- The $k_{FWD-Static-Corr}$ values were on average about 1.3 times lower under cracked panels than under uncracked panels. The COV of the $k$ values were higher under the cracked panels (38%) than under the uncracked panels (23%).
- The $k_{FWD-Static-Corr}$ values were on average about 1.1 times lower in cut areas than in fill areas. The COV of the $k$ values were higher in the cut areas (31%) than in the fill areas (21%).
- There was no statistically significant difference in the I values between the cracked and uncracked panels and the cut and fill areas. The I values were all very low ($\leq 1 \mu m$). I $> 5 \mu m$ is typically considered a trigger value suggesting void beneath the pavement.
- The joint LTE at all panels was relatively high (> 91%) and there was no statistically significant difference between the cracked and the uncracked panels and the cut and fill areas.
REFERENCES


